A fire resistance assessment case history

G.C. and M.E. Giuliani

In general

A structure subjected to fire must be safe for the time necessary for the escape of the people and for the safe operation of the rescue and fire brigade; the verification is based on the fulfilment of the following conditions for the total time necessary for the escape of the people and for the safe operation of the fire brigade:

- **R** structural resistance
- **E** structure and pavement smoke tightness
- **I** isolation or limit of the temperature of the floor above the fire

These conditions are verified taking into account the temperature versus time evolution by means of:

- for the **R** condition: a step by step analysis of the temperature in the ambient and in the structural elements, followed by the verification of the load bearing capacity resulting from the reduction of the material mechanical parameters.
- a verification of the existence of a structure capable of satisfying the **E** and **I** conditions

The assessing of the structure is related to the actual use of the building and to the relevant amount of possibly combustible materials; therefore the calculations are based on the temperature development induced by a real fire in a closed space, which is bound by the floor, the ceiling and the edge partitions and connected to the open air through the side openings

The standard fires, recommended by codes for the structural design or by the national laws, show a continuous increase of temperature which, because of the correspondent reduction of the strength of the materials, limits the time of the load bearing capacity of the structure.

On the contrary, the real fires can evolve from the amount of possibly burning material and from the oxygen available in the space or drawn in through the side openings and therefore always have a decay phase.

In the figure 1 the differences between the temperature evolutions of the above said fires are represented.

In detail

In many cases, recent shopping malls are based on large column grid lines and are developed over two or three stories; a steel-concrete composite structure is a very efficient solution for the floors.

Because of the appealing contrast between the elegant exhibited goods and the high technical aspect of the in sight structure, the application of any fire passive protection spoils the architectural results and only sprinkler system are accepted (see figure 2).

Therefore the “naked” structure fire resistance has to be assessed for the real operating conditions.

In the annexe A of the Euro Code 1, the temperature evolution developed by the natural fire is defined as depending on the fuel amount and on the ventilation conditions given by the compartment geometry, the boundary thermal characteristics and the openings...

Other parameters, defined in the annex E of the Euro Code 1, are related to the danger of fire activation as per the table E1 and to the function of active fire fighting measures, as per the table E2.

A case history

The fire resistance assessment is developed for the composite steel-concrete floor of a building which is composed of main beams continuous over 8.00 m spans and of secondary ones also continuous over 16.00 m spans (see figure 3).

The design value of the fire load $q_{f,d}$ is defined as:

$$q_{f,d} = q_{f,k}\, m\, \delta_{q1}\, \delta_{q2}\, \delta_n \ (\mathrm{MJ/m^2})$$

where

$m = 0.80$ is the combustion factor of the material

δ_{q1} = is a factor taking into account the fire activation risk due to the size of the compartment

δ_{q2} = is a factor taking into account the fire activation risk due to the type of occupancy

$\delta_n = \prod_{i-1}^{10} \delta_{ni} = 0.237$ is a factor taking into account the different active fire fighting measures (sprinkler, detection, automatic alarm transmission, firemen). These active measures are generally imposed for life safety reason.

$q_{f,k}$ $(\mathrm{MJ/m^2})$ is the characteristic fire load density per unit floor area

The temperature raising depends on the kind of the fire which can occur:

- the general fire which is controlled by the ventilation
- the local fire which is controlled by the fuel amount

each one of the above said conditions yields a completely different result. (figures 7 and 8).

The temperature evaluation was performed for two basic fire conditions:

- local fire in a department store bounded by wood shelves; this condition yields a fire development governed by the fuel and is more stringent than the following one
- general fire in the total floor surface bounded by masonry walls with few openings; this condition occurs as a consequence of the local fire and yields a fire development governed by the ventilation.

For the examined case history, according to the site and the building features, the following parameters were used for the calculations taken from tables E1 and E2 of EC1.

δ_{q1} = 1,60	is a factor taking into account the fire activation risk due to the size of the compartment
δ_{q2} = 1,00	is a factor taking into account the fire activation risk due to the type of occupancy
δ_{n1} = 0,61	automatic water extinguishing system
δ_{n2} = 0,87	independent water supplies
δ_{n3} = 1,00	automatic fire detection & alarm – by heat
δ_{n4} = 0,73	automatic fire detection & alarm – by smoke
δ_{n5} = 0,87	automatic alarm transmission to fire brigade
δ_{n6} = 1,00	on site fire brigade
δ_{n7} = 0,78	off site fire brigade
δ_{n8} = 0,90	safe access routes
δ_{n9} = 1,00	fire fighting devices
δ_{n10} =1,00	smoke exhaust system

Because of the above said parameters the fire can be by fuel or by ventilation controlled; the growth rate of the fire was selected as fast (t_{lim}=15 minutes) in all the calculations.
The strategy adopted for the fire resistance assessment was based on the calculation of the maximum amount of possibly burning material which can be stored in the compartment having the internal height **h**=4.50 m ; the goods on show are mainly composed of cellulosic clothes and tissues having a specific calorific value $\mathbf{H_u}$=20.00 MJ/kg (the same parameter for the wood is 17.5) and a combustion factor **m**=0.80.
Two different scenarios were used for the fires controlled by the fuel amount or by the ventilation; the growing rate of the fire was selected as fast with the corresponding limit time t_{lim}=0.25 hours.
The relevant parameters are indicated in the following table:

fire controlled by $\longrightarrow$		**fuel**	**ventilation**
mass of cellulosic material	$\mathbf{m_f}$ (kg/m^2)=	15,00	20,00
resultant fire characteristic design load	$\mathbf{q_{fk}}$ (MJ/m^2) =	300,00	400,00
design value of the fire load density	$\mathbf{q_{fd}}$ (MJ/m^2) =	90,86	121,14
reference area for the natural fire	$\mathbf{A_F}$ (m^2) =	714,24	5120,00
perimeter area	$\mathbf{A_S}$ (m^2) =	513,00	1296,00
opening area	$\mathbf{A_v}$ (m^2) =	46,48	50,40
total area of the enclosure	$\mathbf{A_T}$ (m^2) =	1941,48	11536,00
boundary of enclosure conductivity	λ (W/m/°K) =	0,10	1,04
boundary of enclosure density	$\boldsymbol{\rho}$ (kg/m^3) **=**	450,00	2000,00
boundary of enclosure specific heat	$\mathbf{c_p}$ (J/kg°K) =	1135,00	1113,00
boundary parameter $\sqrt{(\rho c \lambda)}$	**b** (J/m^{2}s$^{1/2}$K)	226,00	1521,00
opening factor $A_v\sqrt{h}$ /A_t)	**O** (m$^{1/2}$)	0,0511	0,0076
boundary parameter $(O/b)^2/(0.04/1160)^2$	Γ	43,06	0,02
resultant fire modified design load (*)	$\mathbf{q_{td}}$ (MJ/m^2) =	33,42	53,77
max fire temperature	$\boldsymbol{\Theta_{max}}$ (°C)	877,23	543,94
time to max temperature (**)	$\mathbf{t_{max}}$ (min)	15,00	32,40

(*) q_{td} (MJ/m^2)=$q_{fd}A_f/A_t$
(**) t_{max}=max(0.0002q_{td}; t_{lim})
In general the fire governed by the ventilation lasts for hours but with temperatures below the steel critical ones.
An interesting remark is given for the above said condition:

- the amount of air necessary for burning one kg of cellulosic material with the relevant calorific value of H_u=20MJ/kg is A_a=0,312H_u+0.65=6.89 kg/kg
- being ρ =1.225 kg/m^3 the specific mass of the air, the specific air volume which is necessary for burning the material is V_a=5.62 m^3/kg

- given the compartment volume $V=23040\ m^3$, the total mass M of the material which can burn by using the inside air is therefore given by $M=V/V_a=4100$ kg, which corresponds to an average distribution of $M/A_F=0.8\ kg/m^2 << m_f$.

The evolution of the temperatures developed by the natural fire is defined by the following function as per the annex **A** of the **Euro code 1**; a decay branch of the temperature curve is taken into account because the temperature increasing of the steel is shifted in time and the maximum value appears during the ambient cooling phase.
The gas temperature versus time is therefore given by the function:
$\Theta_g=20+1325(1\text{-}0.324e^{-0.2\tau}1\text{-}0{,}204\ e^{-1.7\tau}\ \text{-}0.472\ e^{-19\tau})$ in which τ=**t**Γ, with **t** expressed in hours.

Loading condition and material mechanical properties concurrent to the fire

- the fire was considered as an exceptional loading condition.
- the fire, originated in the floors, was considered as a local hazard.
- the material resistance safety factors were reduced accordingly to the temperature.
- the loading safety factors were reduced also $\boldsymbol{g_g}$ =1.0, $\boldsymbol{g_q}$=1.0 and the design live load **q** was reduced by a participation factor $\boldsymbol{\psi_q}$ =0.7 related to the use of the area.

Structural analysis procedure

A step by step versus time procedure for both the cases was used for calculating the temperatures of the fire and of the materials, for determining the correspondent resistance.

- set time to 0s
- increment time 5s
- calculate the temperature of the standard fire versus time
- calculate the correspondent temperature of the steel according to the mass / exposed surface ratio of the beams and of the material and fire parameters
- calculate the resistance of the steel and of the concrete correspondent to the temperature
- for the mid span and the support sections calculate the ultimate resistance according to the schemes of figure 5
- perform an elastic-plastic structural analysis taking into account the action redistribution between the most exposed sections towards the ones subjected to a lower temperature according to the scheme of figure 6: at the ultimate state, the mid span moment capacity is given by $M_{us}=M_{1R}+M_{2R}/2$ and the maximum resisted load is $\mathbf{q_u}=8M_{us}/L^2$
- verify the resistance to the vertical shear at supports and the studs subjected to the horizontal shear
- compare the structure resistance **R** with the action **A** of the supported load
- if **R>A** start a new iteration ; if not, the time resistance is determined, because, due to the exhaustion of its redistribution capability, a part of or the whole structure is transformed into a mechanism.

In the examined case history, because of the limited stiffness of the columns and of the sliding supports at the relevant bases, the axial forces introduced in the beams by the constraint of the thermal strains were not crucial for the ultimate bending resistance.

Calculation of the steel temperatures

The calculation of the temperatures takes into account the geometry of the structure:

- the bottom flange of a beam is directly hit by the fire
- the web and the upper flange are in shadow from the flame radiating effect and therefore are subjected to a lower temperature
- the top of the reinforced concrete slab and the relevant reinforcement are subjected to a much lower temperature

The following relations, taken from Euro code 4, were used for the calculation of the temperature of the structural steel.

$$\Delta\theta_{a.t} = k_{shadov}\left(\frac{1}{c_a\rho_a}\right)\left(\frac{A_i}{V_i}\right)h_{net}\Delta t$$

where

k_{shadov}	is a correction factor for the shadow effect	
c_a	is the specific heat of steel	(J/kgK)
ρ_a	is the density of steel	(kg/m^3)
A_i	is the exposed surface area of the part i of the steel cross-section per unit length	(m^2/m)
A_i/V_i	is the section factor (m^{-1}) of the part i of the steel cross-section	
$\Theta_{a,t}$	is the steel temperature at time t (°C) supposed to be uniform in each part of the steel cross-section	
Δt	is the time interval	(sec)

The shadow effect was determined from:

$$k_{shadow} = 0{,}9\left(\frac{e_1 + e_2 + 1/2 \cdot b_1 + \sqrt{h_w^2 + 1/4 \cdot (b_1 - b_2)^2}}{h_w + b_1 + 1/2 \cdot b_2 + e_1 + e_2 - e_w}\right)$$

With e_1, b_1, e_w, h_w, e_2, b_2 and cross sectional dimensions

Because of the different section sizes and of the continuity conditions (see figure 4), the verifications were effected for any single element of the structure; the results for the edge bay of a secondary beam over the 16.00 m span are illustrated in the figures 7 and 8.

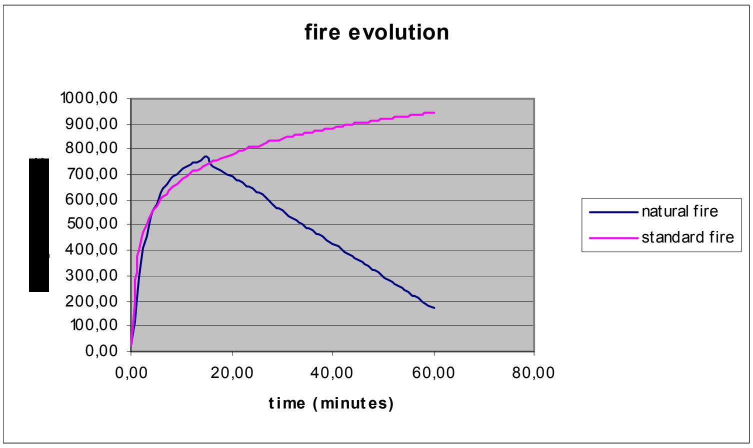

figure 1 - Temperature evolutions of the standard and of the natural fire

figure 2 - The “naked” steel structure

figure 3 – The composite steel-concrete floor with column at 8.00 by 16.00 m centres, main and secondary beams continuous

figure 4 – The two way beam to column moment resisting connection

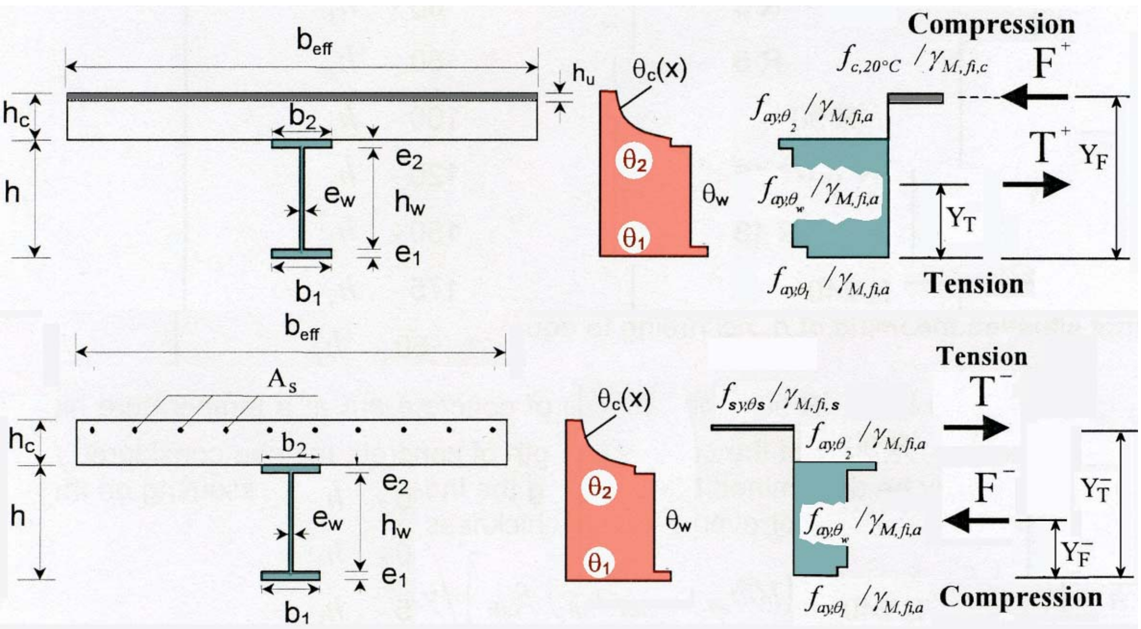

figure 5 – Plastic verification of the composite sections for sagging and hogging moments respectively

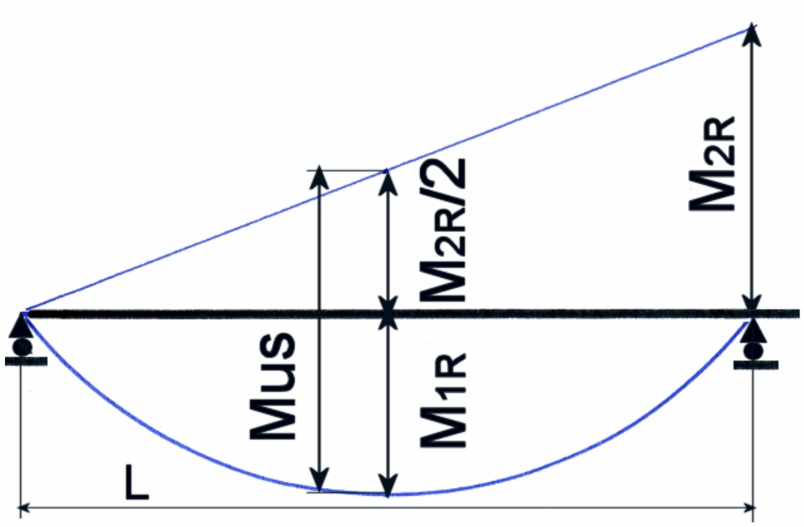

figure 6 – Scheme of the moment redistribution for an edge bay of the continuous beam

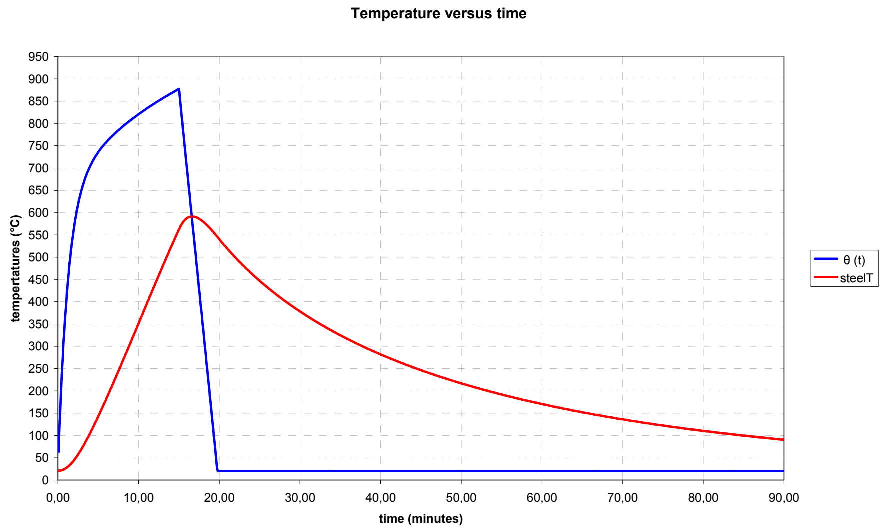

figure 7 - Calculated temperatures at the secondary beam mid span for the fuel controlled fire

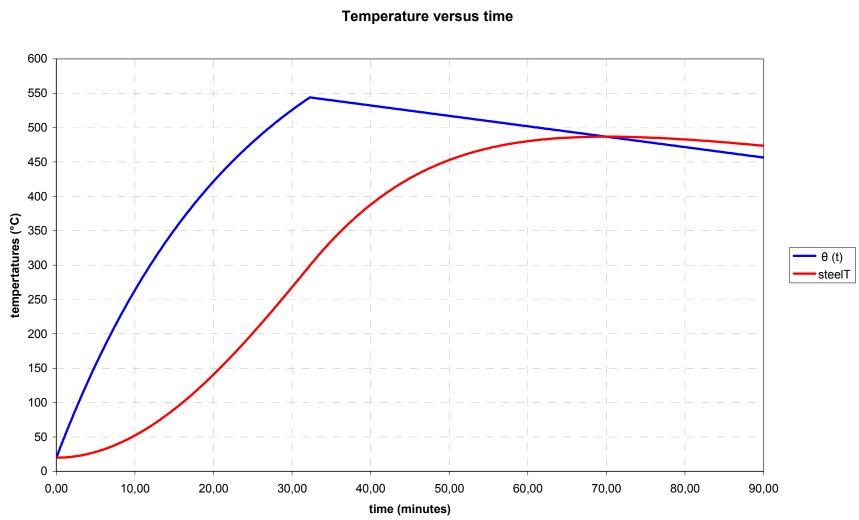

figure 8 - Calculated temperatures at the secondary beam mid span for the ventilation controlled fire